CBC Engineers & Associates

June 30, 2009

Clark County Engineer's Department
4075 Laybourne Road
Springfield, OH 45505

Attn: Paul W. DeButy, P.E.
Deputy, Engineering and Planning

Re: Geotechnical Engineering Investigation for a Proposed Replacement Bridge for an Existing Truss Bridge on Redmond Road, Springfield, Ohio; CBC Report No. 10731-1-0609-02

Gentlemen:

We are pleased to submit our report of the geotechnical engineering investigation for the above-referenced project. The purpose of this study was to provide an evaluation of the physical characteristics of the soil strata and net allowable bearing capacities at the locations tested. Also noted are other conditions that might affect the design and/or construction of the proposed bridge foundations based on the results of the testing.

For your convenience, the samples collected that were not used to perform the laboratory tests will be kept in our office for a period of three months. If you have any questions, or if we can help you in any way, please call us.

Respectfully submitted,

CBC Engineers & Associates, Ltd.

STATE OF OHIO
DAVID C. COWHERD
30371
REGISTERED PROFESSIONAL ENGINEER

David C. Cowherd, M.S., P.E.
CEO and Chief Engineer

DCC/smh
3-Client
1-File

CBC Engineers & Associates, Ltd.
125 Westpark Road Centerville, Ohio 45459
Phone: 937-428-6150 FAX: 937-428-6154 Email: cbceng@cbceng.com

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SECTION I

TEXT

1.0 INTRODUCTION

Authorization to proceed with this investigation was given by Mr. Paul DeButy of Clark County Engineer's Department. Work was to proceed in accordance with CBC Engineers & Associates, Ltd. Quotation No. 09-288-02 (R1) dated June 15, 2009, and the terms and conditions of the contract attached thereto.

The proposed bridge foundation is located on Redmond Road in Springfield, Ohio. A Vicinity Map is presented in Figure 1 in Section III of this document.

2.0 WORK PERFORMED

2.1 FIELD WORK

Two (2) borings were made in the relative positions shown on the Boring Location Plan (Figure 2) in Section III. The boring logs and resulting data are also included in Section III. The borings were made with a truck-mounted boring rig using hollow-stem augers and employing standard penetration resistance methods (ASTM D-1586, which includes 140-pound hammer, 30-inch drop, and two-inch-O.D. split-spoon sampler) at maximum depth intervals of five feet or at major changes in stratum, whichever occurred first. The disturbed split-spoon samples were visually classified, logged, sealed in moisture-proof jars, and taken to the CBC Engineers & Associates, Ltd. laboratory for study. The depths where these "A"-type split-spoon samples were collected are noted on the boring logs.

2.2 LABORATORY WORK

One ODOT classification test was performed in accordance with ASTM D422. A typical sample of the soil at the level of the pile cap was classified according to the ODOT Classification System.

Twenty-Four (24) natural moisture content determinations were made in accordance with ASTM D-4643. The results of these tests are tabulated in Table 1 as follows, and are also included in Section III of this report:

TABLE 1

RESULTS OF NATURAL MOISTURE CONTENT TESTS (ASTM D-4643

BORING NO.	DEPTH INCREMENT, (FT.)	NATURAL MOISTURE CONTENT, %
B-1	1.0 – 2.5	6.7
B-1	3.5 – 5.0	15.0
B-1	6.0 – 7.5	26.7
B-1	8.5 – 10.0	25.4
B-1	13.5 – 15.0	10.2
B-1	18.5 – 20.0	9.2
B-1	23.5 – 25.0	10.0
B-1	28.5 – 30.0	10.0
B-1	33.5 – 35.0	10.8
B-1	38.5 – 40.0	10.6
B-1	43.5 – 45.0	10.6
B-1	48.5 – 50.0	9.2
B-2	1.0 – 2.5	13.8
B-2	3.5 – 5.0	14.2
B-2	6.0 – 7.5	17.2
B-2	8.5 – 10.0	14.8
B-2	13.5 – 15.0	10.2
B-2	18.5 – 20.0	10.4
B-2	23.5 – 25.0	9.9
B-2	28.5 – 30.0	9.9
B-2	33.5 – 35.0	10.9
B-2	38.5 – 40.0	10.4
B-2	43.5 – 45.0	10.6
B-2	48.5 – 50.0	4.9

3.0 SOIL CONDITIONS AND GROUNDWATER LEVELS

The site is overlain by fill for the roadway approaches to the bridge. The fill consists of clay, silt, sand, and gravel, and extends to depths of 8'. Below the fill and extending to a depth of 12' is a stratum of brown clay and silt with some sand. This stratum is alluvium. The standard penetration tests in this stratum are 9 to 12. Below this stratum and extending to the bottom of the borings at 50' is a stratum of gray silt with some sand, some clay and a trace of gravel. The standard penetration tests in this stratum vary between 22 and 48.

Groundwater observations were made during the drilling operations (by noting the depth of water on the drilling tools) and in the open boreholes following withdrawal of the drilling augers. Free groundwater was not encountered at the time of drilling activities. However, it should be noted that short-term water level readings are not necessarily a reliable indication of the groundwater level and that significant fluctuations may occur due to variations in rainfall and other factors. For specific questions on the soil conditions, please refer to the individual boring logs in Section III.

Based on the encountered soil conditions at the project site, the site classification was determined to be "Site Class C" per the Ohio Building Code. A Class "C" soil is described as a soil which has an average standard penetration value of greater than 50 for the top 100 feet of soil profile. The following statement is made in the building code "where site-specific data are not available to a depth of 100', appropriate soil properties are permitted to be estimated by the registered geotechnical engineer preparing the soils report, based on known geologic conditions". In general, the standard penetration values below 50' in the till in the area are in excess of 100, therefore foundations can be designed for a site classification "C". In addition, a S_{DS} coefficient of 0.16g was calculated, and a S_{D1} coefficient of 0.1g were also calculated for design based on the building code.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 PROJECT DESCRIPTION

The structure to be constructed at this site is a single-span pre-stressed concrete box beam on capped-pile abutments. No detailed structural loading information is available at this time. The following recommendations are based on this information. Consequently, if the above information is incorrect or if changes are made, CBC Engineers & Associates, Ltd. should be notified so that the new data can be reviewed.

4.2 FOUNDATIONS

We understand it is desirable to use a driven pile foundation for support of the bridge. We do not at this time know the proposed depth of the pile cap. The fill and underlying alluvium extending to a depth of 12' should be assumed to have no support value for piles. The gray till encountered at 12' has a working friction value of 2000 psf of area of pile in contact with the soil, and an end bearing capacity of 20,000 psf for piles below 30' in depth. The piles should be designed for these values. Specifications for driven piles are included in Part II of this report.

An alternative to driven piles would be drilled piers. Drilled piers supported on the gray till can be designed with a friction value of 2,000 psf for the area of piers in contact with the gray till and an end bearing capacity of 12,000 psf for piers below 20'. Drilled piers should be placed in accordance with the Specifications for Drilled Piers included in Part II of this report.

A third alternative would be to place spread foundations on the gray till at a depth of 12' and design for a bearing capacity of 8,000 psf.

The soils information needed for the evaluation of scour is as follows:

$D_{85} = 1$ mm

$D_{50} = 0.065$ mm

$D_{15} = 0.004$ mm

We understand the actual scour analysis is being done by others.

4.2.1 LATERAL AND UPLIFT FORCES ON PILES

Lateral forces on the foundation elements can be resisted by passive lateral earth pressures against the opposite vertical face of the foundation. An allowable resisting passive earth pressure coefficient of 3 can be used for design purposes. The passive resistance should only be used for that portion of the foundation located at a depth greater than 12 feet beneath the existing grade. A factor of safety of 1.5 relative to the lateral capacity should be used in design.

4.2.2 LATERAL EARTH PRESSURES ON BELOW GRADE WALLS

The magnitude of lateral earth pressure against subsurface walls (such as abutment and wingwalls) is dependent on the method of backfill placement, the type of backfill soil, drainage provisions and whether or not the wall is permitted to yield during and/or after placement of the backfill. When a wall is held rigidly against horizontal movement, the lateral pressure against the wall is greater than the "active" earth pressure that is typically used in the design of free-standing retaining walls. Therefore, rigid walls should be designed for higher, "at-rest" pressures (using an at-rest lateral earth pressure coefficient, K_o), while yielding walls can be designed for active pressures (using an active lateral earth pressure coefficient, K_a).

For use in these computations, a total soil unit weight of 130 lbs/cu. ft. should be used. For below-grade walls, a coefficient of earth pressure at-rest (K_o) of 0.45 and a coefficient of "active" earth pressure of 0.30 are recommended, provided a well-graded granular material is used for backfill. Also, a passive earth pressure coefficient of 3.0 should be used in design.

It is recommended that the static weight per axle of equipment utilized for the compaction of the backfill materials not exceed 2 tons per axle for non-vibratory equipment and 1 ton per axle for vibratory equipment. All heavy equipment, including compaction equipment heavier than recommended above, should not be allowed closer to the wall (horizontal distance) than the vertical distance from the backfill surface to the bottom of the wall. If it is desired to use heavier compaction equipment adjacent to the below grade wall, it is recommended that this office be contacted to determine the resulting earth pressures.

4.2.3 FOUNDATION EXCAVATIONS

Each foundation excavation should be inspected to insure that all loose, soft or otherwise undesirable material is removed and that the foundation will bear on satisfactory material.

If pockets of soft, loose or otherwise unsuitable material are encountered in the footing excavations and it is inconvenient to lower the footings, the proposed footing elevations may be re-established by backfilling after the undesirable material has been removed. The undercut excavation beneath each footing should extend to suitable bearing soils and the dimensions of the excavation base should be determined by imaginary planes extending outward and down on a 1 (vertical) to 1 (horizontal) slope from the base perimeter of the footing as illustrated in Figure 3 in Section III. The entire excavation should then be refilled with a well-compacted engineered fill, or lean concrete (please note that the width of the lean concrete zone should be equal to or wider than the width of the overlying footing element). Special care should be exercised to remove any sloughed, loose or soft materials near the base of the excavation slopes. All Federal, State, and Local regulations should be strictly adhered to relative to excavation side-slope geometry.

5.0 SLOPE CONSIDERATIONS

A detailed slope stability analysis is beyond the scope of this study. However, it is, recommended that fill slopes less than 10 feet in height be designed for slopes not steeper than 2.5 (horizontal) to 1 (vertical). For any fill greater than 10 feet in height, it is recommended that slopes be not steeper than 3 (horizontal) to 1 (vertical).

In general, temporary cut slopes of 2 (horizontal) to 1 (vertical) should remain stable during a reasonable construction period provided they are not higher than about 10 feet and are not subjected to excessive vibration from construction equipment and are protected from surface erosion. The need for temporary bracing of utility trenches should be anticipated. In general, any permanent cut slopes should be no steeper than about 3 (horizontal) to 1 (vertical).

6.0 CONSTRUCTION DEWATERING

At the time of our investigation, the free groundwater level was noted to be generally below the anticipated footing depth. However, it is likely that some seepage into foundation excavations will occur, depending on the seasonal conditions. Excavations which intercept saturated, discontinuous sand and gravel lenses or other wet granular zones may encounter significant quantities of free groundwater. It is anticipated that any such seepage can be intercepted by open sumps from which the water can be pumped. However, care must be exercised when pumping from sumps that extend into silts or other granular soils, as a general deterioration of the bearing soils and a localized "quick" condition could result. If significant groundwater influxes are noted within the excavations, other dewatering techniques should be determined at the time of construction.

7.0 SOIL SWELLING POTENTIAL

Based upon the laboratory tests performed for this study and the mineralogy of typical soils from the general vicinity of the project site, no significant soil swelling is anticipated. To our knowledge, there are no instances of problems associated with soil swelling in the project vicinity.

8.0 LIQUEFACTION

When certain soils (generally only granular soils) below the groundwater table are subjected to dynamic loads, such as those produced by earthquakes, a sudden increase in pore water pressure occurs as the result of shearing of the soil particles passed one another. In extreme cases, when these shear induced pore water pressures exceed the strength of the soil, the soil strength can reduce to zero thereby resulting in a phenomenon known as "liquefaction." Conditions at this site have been examined to determine the likelihood for liquefaction of the natural soils during earthquake ground motions.

Soil type, relative density, initial confining pressure (i.e., the depth of the potentially liquefiable soil below the ground surface) and the magnitude of potential ground motions are the most important factors in determining the liquefaction potential of a soil mass. It is generally agreed that saturated, relatively loose (with blow counts or "N" values typically less than about 13) in the upper 50 feet or so are most susceptible to liquefaction.

Clayey soils are generally considered to be non-vulnerable to liquefaction. It is, therefore, concluded that liquefaction (or any significant loss of strength) of the soils underlying the project site during earthquake ground motions is extremely unlikely. To our knowledge, there are no recorded cases of liquefaction of subsurface materials similar to those at this project site. Therefore, no special design measures relative to soil liquefaction appear to be warranted.

9.0 BURIED UTILITY PIPES

Excavations for buried utility pipelines should follow the guidelines set forth previously in this report. Depending on the pipeline material, a minimum thickness of at least 0.5 foot of select fine-grained granular bedding material should be used beneath all below-grade pipes, with a minimum cover thickness of at least 3 feet to afford an "arching" effect and reduce stresses on the pipe. The cover thickness may be reduced if the external loading condition on the pipe is relatively light or if the pipe is designed to withstand the external loading condition. It is not recommended that "pea-gravel" or other "open-work" aggregates be used for trench backfill since these materials are nearly impossible to compact and have a tendency to pond water within their interstices.

10.0 DRAINAGE

Adequate drainage should be provided at the site to minimize any increase in moisture content of the foundation soils. The exterior grade (including all parking areas) should be sloped away from all facility structures to prevent ponding of water.

11.0 CLOSURE

11.1 BASIS OF RECOMMENDATIONS

The evaluations, conclusions, and recommendations in this report are based on our interpretation of the field and laboratory data obtained during the exploration, our understanding of the project and our experience with similar sites and subsurface conditions. Data used during this exploration included, but were not necessarily limited to:

- two (2) exploratory borings performed during this study,
- observations of the project site by our staff,
- results of the laboratory soil tests,
- site plans and drawings furnished by Clark County Engineer's Department,
- limited interaction with Paul DeButy of Clark County Engineer's Department; and
- published soil or geologic data of this area.

In the event that changes in the project characteristics are planned, or if additional information or differences from the conditions anticipated in this report become apparent, CBC

Engineers & Associates, Ltd., should be notified so that the conclusions and recommendations contained in this report can be reviewed and, if necessary, modified or verified in writing.

11.2 LIMITATIONS OF STUDY/RECOMMENDED ADDITIONAL SERVICES

The subsurface conditions discussed in this report and those shown on the boring logs represent an estimate of the subsurface conditions based on interpretation of the boring data using normally accepted geotechnical engineering judgments. Although individual test borings are representative of the subsurface conditions at the boring locations on the dates shown, they are not necessarily indicative of subsurface conditions at other locations or at other times.

Regardless of the thoroughness of a subsurface exploration, there is the possibility that conditions between borings will differ from those at the boring locations, that conditions are not as anticipated by designers, or that the construction process has altered the soil conditions. As variations in the soil profile are encountered, additional subsurface sampling and testing may be necessary to provide data required to re-evaluate the recommendations of this report. Consequently, after submission of this report it is recommended that CBC Engineers & Associates, Ltd. be authorized to perform additional services to work with the designer(s) to minimize errors and omissions regarding the interpretation and implementation of this report.

Prior to construction, we recommend that CBC Engineers & Associates, Ltd.:

- work with the designers to implement the recommended geotechnical design parameters into plans and specifications,
- consult with the design team regarding interpretation of this report,
- establish criteria for the construction observation and testing for the soil conditions encountered at this site; and
- review final plans and specifications pertaining to geotechnical aspects of design.

During construction, we recommend that CBC Engineers & Associates, Ltd.:

- observe the construction, particularly the site preparation, fill placement, and foundation excavation or installation,

- perform in-place density testing of all compacted fill,
- perform materials testing of soil and other materials as required; and
- consult with the design team to make design changes in the event that differing subsurface conditions are encountered.

If CBC Engineers & Associates, Ltd. is not retained for these services, we shall assume no responsibility for construction compliance with the design concepts, specifications or recommendations.

11.3 WARRANTY

Our professional services have been performed, our findings obtained and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. No other warranty, express or implied, is made.

While the services of CBC Engineers & Associates, Ltd. are a valuable and integral part of the design and construction teams, we do not warrant, guarantee, or insure the quality or completeness of services provided by other members of those teams, the quality, completeness, or satisfactory performance of construction plans and specifications which we have not prepared, nor the ultimate performance of building site materials.

11.3.1 SUBSURFACE EXPLORATION

Subsurface exploration is normally accomplished by test borings, although test pits are sometimes employed. The method of determining the boring location and the surface elevation at the boring is noted in the report, and is presented on the Boring Location Plan or on the boring log. The location and elevation of the boring should be considered accurate only to the degree inherent with the method used.

The boring log includes sampling information, description of the materials recovered, approximate depth of boundaries between soil and rock strata and groundwater data. The boring log represents conditions specifically at the location and time the boring was made. The boundaries between different soil strata are indicated at specific depths; however, these depths are in fact approximate and are somewhat dependent upon the frequency of sampling (The transition between soil strata is often gradual). Free groundwater level readings are made at the times and under conditions stated on the boring logs (Groundwater levels change with time and season). The borehole does not always remain open sufficiently long enough for the measured water level to coincide with the groundwater table.

11.3.2 LABORATORY AND FIELD TESTS

Laboratory and field tests are performed in accordance with specific ASTM standards unless otherwise indicated. All determinations included in a given ASTM standard are not always required and performed. Each test report indicates the measurements and determinations actually made.

11.3.3 ANALYSIS AND RECOMMENDATIONS

The geotechnical report is prepared primarily to aid in the engineering design of site work and structural foundations. Although the information in the report is expected to be sufficient for these purposes, it is not intended to determine the cost of construction or to stand alone as a construction specification.

Our engineering report recommendations are based primarily on data from test borings made at the locations shown on a boring location plan included in this report. Soil variations may exist between borings and these variations may not become evident until construction. If significant variations are then noted, the geotechnical engineer should be contacted so that field conditions can be examined and recommendations revised if necessary.

The geotechnical engineering report states our understanding as to the location, dimensions and structural features proposed for the site. Any significant changes in the nature, design, or location of the site improvements MUST be communicated to the geotechnical engineer such that the geotechnical analysis, conclusions, and recommendations can be appropriately adjusted. The geotechnical engineer should be given the opportunity to review all drawings that have been prepared based on their recommendations.

11.3.4 CONSTRUCTION MONITORING

Construction monitoring is a vital element of complete geotechnical services. The field engineer/inspector is the owner's "representative" observing the work of the contractor, performing tests as required in the specifications, and reporting data developed from such tests and observations. The field engineer or inspector does not direct the contractor's construction means, methods, operations or personnel. The field inspector/engineer does not interfere with the relationship between the owner and the contractor and, except as an observer, does not become a substitute owner on site. The field inspector/engineer is responsible for his own safety but has no responsibility for the safety of other personnel at the site. The field inspector/engineer is an important member of a team whose responsibility is to watch and test the work being done and report to the owner whether that work is being carried out in general conformance with the plans and specifications.

11.3.5 GENERAL

The scope of our services did not include an environmental assessment for the presence or absence of hazardous or toxic materials in the soil, surface water, groundwater or air, on, within or beyond the site studied. Any statements in the report or on the boring logs regarding odors, staining of soils or other unusual items or conditions observed are strictly for the information of our client.

To evaluate the site for possible environmental liabilities, we recommend an environmental assessment, consisting of a detailed site reconnaissance, a record review, and report of findings. Additional subsurface drilling and samplings, including groundwater sampling, may be required. CBC Engineers & Associates, Ltd. can provide this service and would be pleased to provide a cost proposal to perform such a study, if requested.

This report has been prepared for the exclusive use of Clark County Engineer's Department for specific application to the proposed bridge foundations (see Figure 1 in Section III of this report). Specific design and construction recommendations have been provided in the various sections of the report. The report shall, therefore, be used in its entirety. This report is not a bidding document and shall not be used for that purpose. Anyone reviewing this report must interpret and draw their own conclusions regarding specific construction techniques and methods chosen. CBC Engineers & Associates, Ltd. is not responsible for the independent conclusions, opinions or recommendations made by others based on the field exploratory and laboratory test data presented in this report.

SECTION II

SPECIFICATIONS

I – DRIVEN TIMBER PILES

1.0 GENERAL

1.1 PROTECTION

1.1.1 Avoid damaging pile by bruising or breaking of wood fibers.

1.1.2 Avoid breaking surface of treated piles.

1.1.3 Do not damage surface of treated piles below cutoff elevation by boring holes or driving nails or spikes into them to support temporary material.

1.2 METHOD OF MEASUREMENT

1.2.1 Supply and installation of piles will be measured in total length of piles accepted and incorporated into work.

1.2.2 Pile shoes and pile head protection are incidental to the supply and installation of the piles.

1.2.3 Preservative treatment is incidental to the supply and installation of piles.

1.2.4 Mobilization and demobilization costs for all equipment will be measured separately.

2.0 PRODUCTS

2.1 MATERIALS

2.1.1 Round wood pile to be in accordance with ASTM D-25 for clean peeled piles with minimum butt size of 12 inches and diameter of tree top (pile toe) related to length for Class "B" timber piles. Order-length of piles to be 50 feet.

2.1.2 Preservative treatment to be in accordance with AWPA M4.

2.1.3 Engineer will be sole judge of quality and dimensions of piles. Remove rejected piles from site of work.

2.2 MISCELLANEOUS MATERIAL

2.2.1 Bolts, nuts, and washers to be in accordance with ASTM A307-82a.

2.2.2 Do not use items fabricated from scrap steel of unknown chemical composition or physical properties.

2.2.3 Hot-dip galvanized bolts, nuts, and washers and, unless otherwise specified, staples, cable clamps, pipe sleeves, spikes, and nails to be in accordance with ASTM A153-82. Other galvanized hardware to be in accordance with ASTM A123-78.

3.0 EXECUTION

3.1 PREPARATION

3.1.1 Protect pile toe using an APF T-9168 shoe, or equivalent.

3.1.2 Protect pile by means of steel straps placed along the length of pile with at least two straps placed within 3 feet of butt of pile.

3.2 INSTALLATION

3.2.1 Keep the pile driving helmet concentric and square with the pile head at all times and the leads in alignment with the pile. Occasionally during the driving, and whenever requested by Engineer, lift off (lighten) helmet from pile to verify that the helmet and leads are not inducing bending stresses into the pile.

3.2.2 Treat exposed ends of cut-off pile with two liberally brushed coats of hot creosote followed by an application of coal tar pitch, allowing sufficient interval between applications to permit absorption.

3.3 PILE CAPS

3.3.1 Supply and install timber pile caps as indicated.

3.3.2 Treat ends of timber in accordance with Clause 3.2.2 of these specifications.

3.4 EQUIPMENT INFORMATION

3.4.1 Prior to commencement of pile installation operation, submit to Engineer for approval, details of equipment for installation of piles.

3.4.2 For impact hammers, give manufacturer's name, type, maximum rated energy and rated energy per blow at normal working rate during easy and at termination driving, mass of striking parts of hammer, mass of driving cap, and type and elastic properties of hammer cushion.

3.4.3 IMPACT HAMMER

3.4.3.1 For final driving of timber piles, provide a hammer capable of delivering to the pile a non-erratic impact load not smaller than one-half of the design axial load of the pile to the pile head at normal working rate.

3.4.3.2 Remedial action due to failure of the Contractor's hammer equipment will be at the Contractor's own expense. Such remedial action may consist of, but need not be limited to, adjustment or replacement of hammer cushion, or of pile cushion, or to adjustment or replacement of hammer.

3.4.4 LEADS

3.4.4.1 Provide leads that will enable the hammer to deliver impacts concentrically and in alignment with the pile longitudinal axis without inducing rocking movements or bending moments in pile.

3.4.4.2 Performance of the leads will be subject to assessment of Engineer. Any remedial action required will be at the Contractor's own expense.

3.5 PREPARATION

3.5.1 Ensure that ground conditions at the pile locations are adequate to support pile driving and loading-test operations (if applicable). Make provision for access and support of piling equipment during performance of work.

3.5.2 Do not commence pile driving before excavation has been completed.

3.5.3 Do not drive piles within embankments until embankment has been placed and compacted to at least bottom elevation of pile cap.

3.6 FIELD MEASUREMENTS

3.6.1 Maintain accurate and daily records of driving for each pile cushion and follower.

3.6.2 Type, make, and rated energy of hammer.

3.6.3 Other installation equipment including details on use of pile cushion and leads.

3.6.4 Pile size and length, location of pile in pile group, and location or designation of pile group.

3.6.5 Time for start and finish of driving pile and sequence of pile driving for piles in group.

3.6.6 Penetration for own weight and own weight and weight of hammer, number of blows per one (1) foot of penetration from start of driving, and penetration per one (1) foot when approaching termination of driving of pile.

3.6.7 Observed stroke and blow rate (blows/minute) of hammer.

3.6.8 Toe elevation upon termination of driving pile and final toe and cut-off elevations upon completion of pile group.

3.6.9 Upon termination of the driving of open-toe pipe piles, record depth from ground surface outside pile to soil surface inside pipe.

3.6.10 Records of restriking.

3.6.11 Other pertinent information, such as interruption of continuous driving, observed pile damage, etc.

3.6.12 Records of elevation of adjacent piles before and after driving of pile.

3.6.13 Record all information on forms provided by Engineer.

3.6.14 Provide Engineer with three copies of the records.

3.6.15 Use driving helmet to protect pile head.

3.6.16 Do not use any loose inserts in the helmet. The Engineer is sole judge of the acceptability of the helmet.

3.6.17 Hold pile securely and accurately in position while driving.

3.6.18 Deliver hammer impacts concentrically and in direct alignment with pile taking care to avoid forcing pile laterally or bending pile. If in the Engineer's opinion, lateral or bending forces unduly affect the pile, the Contractor must stop and rectify the situation at his own expense and to the satisfaction of the Engineer.

3.6.19 Reinforce pile heads, if and as necessary.

3.6.20 Advance pile to toe elevation as indicated on drawing and penetration resistance specified in Clause 3.8.1.

3.6.21 Do not drive piles within a radius of 25 feet of concrete which has been in place for a time shorter than 3 days unless authorized by Engineer.

3.6.22 Restrike piles which have settled or heaved during driving of adjacent piles. No additional compensation will be made for pile restruck due to such settlement or heave.

3.6.23 Remove loose and displaced material from around pile after completion of driving, and leave clean, solid surfaces to receive concrete.

3.6.24 Provide sufficient length above cut-off elevation so that part damaged during driving is cut off. Cut off piles neatly and squarely at elevations indicated.

3.6.25 After driving, pile must be accessible for inspection of integrity through the full length of pile.

3.6.26 Remove cut-off lengths from site on completion of work.

3.7 OBSTRUCTIONS

3.7.1 Where obstructions are encountered that results in sudden, unexpected change in penetration resistance and deviation from specified tolerances, the Contractor may be required to perform one or all of the following:

3.7.1.1 Removal of obstruction.

3.7.1.2 Extraction, repositioning, and redriving.

3.7.1.3 Addition of extra piles.

3.7.2 If, in the opinion of Engineer, work done as per Clause 3.7.1 could not have been reasonably anticipated by the Contractor, additional compensation for work done will be considered for payment.

3.8 DESIGN LOAD

3.8.1 The required design load of each timber pile is 25 tons (50,000 lbs) and should be verified in the field using the ENR formula as follows:

$$R = \frac{2WH}{s + 0.1}$$

Where:

R = Resistance for design, lbs. = 50,000 lbs. (in this case)
W = Weight of hammer, lbs.
H = Height of hammer fall, ft.
s = set of pile, inches/blow
0.1 = Elastic losses in cap, pile, and soil for a steam/air hammer

Axial load capacities should be modified in the field using this formula, and additional pile lengths added as required.

Please note that a factor of safety is incorporated in the ENR formula.

3.9 PENETRATION RESISTANCE

3.9.1 Installation of each pile will be subject to approval of Engineer, who will be sole judge of acceptability of pile with respect to penetration resistance at end-of-initial-driving as well as at restriking, to depth of penetration, or to other penetration criteria. Engineer to approve final penetration resistance of all piles prior to removal of pile driving equipment from site.

3.9.2 Prior to taking final penetration resistance, drive piles without interruption for a sufficient interval to break or prevent development of soil set-up.

3.9.3 Drive each pile to a minimum toe elevation of -47 feet MSL as indicated.

3.9.4 When required by Engineer, restrike piles to the same criterion as applied in initial driving (Clause 3.8.1). No additional compensation will be made for restriking.

3.10 TOLERANCES

3.10.1 Pile heads at cut-off elevation to be within 3 inches of locations indicated as measured immediately after termination of initial driving and 6 inches as measured after all piles have been driven. To achieve pile installation within tolerances specified, the Contractor may have to resort to using temporary bracing and templates.

3.10.2 Pile rotation to be limited to 3 degrees.

3.10.3 Maintain piling within tolerances specified throughout execution of work.

3.10.4 If in the opinion of Engineer piles are placed beyond tolerances specified, the Contractor may be required to remove such piles and install new piles to the specified tolerances at his own expense.

3.11 DAMAGED OR DEFECTIVE PILES

3.11.1 Engineer will reject any pile found to be defective or damaged.

3.11.2 Remove rejected pile and replace with a new and, if necessary, longer pile.

3.11.3 No extra compensation will be made for removing and replacing or other work made necessary through rejection of a defective pile.

3.12 LOADING TEST

3.12.1 Provide static loading test on pile(s) as selected by Engineer and at any time during performance of work. Static load tests will be used to verify the results of the ENR formula outlined in Clause 3.8.1. Static load tests shall conform with the procedures outlined by D 1143-81 for compressive load tests, and ASTM D-3689-90 for tensile load tests.

3.12.2 Failure of loading test to show satisfactory performance due to inadequate equipment and/or arrangement will result in rejection of test and testing of additional piles.

II – DRILLED PIER INSTALLATION

1.0 DRILLING PROCEDURE

1.1 Drilled piers will be installed with large caisson drill rigs capable of torque and crowd forces sufficient to install drilled piers at the project site given the in-situ soil conditions.

1.2 The drill rig kelly bar and auger will be carefully and accurately placed over the centerline of the drilled pier. The Contractor is responsible for providing necessary surveying to verify drilled pier location before, during, and after the drilled pier installation.

1.3 The augers are advanced downwards as they are rotated such that drilling of the soil mass is efficiently accomplished. Depending on the subsurface conditions, and the requirements for the given project, a temporary steel casing should be installed at this time to preclude caving of the soil and/or broken rock mass being penetrated.

2.0 CASING INSTALLATION

2.1 The casing will be checked for centerline accuracy and plumbness by the Contractor's survey crew. During casing installation, the Contractors survey crew will verify alignment with instruments. If plumbness and alignment are not within tolerance as determined by the Contractors survey crew, the casing will be extracted and re-aligned as necessary.

2.2 The drill rig will remove soil and bedrock material from within the casing to the drilled pier design tip elevation. A steel casing, or "Sonotube" shall be inserted into the borehole to preclude cave-ins and/or instability in the borehole.

2.3 The bearing surface within the drilled pier will be inspected by a registered Professional Engineer prior to being approved for structural concreting.

3.0 INSTALLATION OF THE REBAR CAGE

3.1 An epoxy coated spiraled reinforcing steel cage will be installed while in the drilled pier borehole.

3.2 To assist in assuring that the reinforcing steel cage does not settle during concrete pumping, a mat of reinforcing steel bars will be installed across the bottom of the reinforcing steel cage perpendicular to the vertical axis of the cage. The exact number of bars will be determined and installed by the Structural Engineer. The number of rebar boots used on the bottom of the cage will also be determined by the Structural Engineer.

3.3 The reinforcing steel cage will be lowered into the drilled pier borehole, while drilled pier spacers are placed at intervals as required by the Structural Engineer. The reinforcing steel cage will be checked for alignment by the Contractors survey crew.

3.4 The crane will remain attached to the reinforcing steel cage while the concrete pump outlet pipe is lowered to just above the bottom of the drilled pier. The concrete pump pipe sections will be welded together to assure that do not separate during pumping.

4.0 CONCRETING OF THE DRILLED PIER

4.1 Concrete pumping may commence once the bearing surface has been approved in accordance with Clause 2.3

4.2 A three inch trash pump will be used to pump slurry and/or water from within the casing and from above the newly pumped concrete.

4.3 The concrete pump outlet pipe will maintain at least ten (10) feet of embedment into the fresh concrete. The concrete level in the casing will be monitored.

4.4 The casing will be completely extracted with the crane and/or vibratory hammer. Caisson clamps on the vibratory hammer (if applicable) will be adjusted to the proper dimension to withdrawal the casing.

4.5 The concrete will be terminated at the top of drilled pier elevation and screeded flat.

4.6 The upper reinforcing steel dowel cage will be lowered into the concrete to the embedment elevation. If necessary, the concrete will be vibrated to assist in placement. Alignment will be verified by the Contractors survey crew and the cage will be sufficiently braced.

SECTION III

BORING LOGS, LAB TESTING RESULTS, & PRINTS

BORING LOG TERMINOLOGY

STRATUM DEPTH

Distance in feet and/or inches below ground surface.

STRATUM ELEVATION

Elevation in feet below ground surface elevation.

DESCRIPTION OF MATERIALS

Major types of soil material existing at boring location. Soil classification based on one of the following systems: Unified Soil Classification System, Ohio State Highway Classification System, Highway Research Board Classification System, Federal Aviation Authority Classification System, Visual Classification.

SAMPLE NO.

Sample numbers are designated consecutively, increasing with depth for each boring.

SAMPLE TYPE

"A" Split spoon, 2" O.D., 1-3/8" I.D., 18" in length.

"B" One of the following:

- Power Auger Sample
- Piston Sample
- Diamond Bit NX: BX: AX:
- Housel Sample
- Wash Sample
- Denison Sample

"C" Shelby Tube 3" O.D. except where noted.

SAMPLE DEPTH

Depth below top of ground at which appropriate sample was taken.

BLOWS PER 6" ON SAMPLER

The number of blows required to drive a 2" O.D., 1-3/8" I.D., split spoon sampler, using a 140 pound hammer with a 30 inch free fall, is recorded for 6" drive increments. (Example: 3/8/9)

"N" BLOWS/FT.

Standard penetration resistance. This value is based on the total number of blows required for the last 12" of penetration. (Example: 3/8/9 $\therefore$ N = 8 + 9 = 17)

WATER OBSERVATIONS

Depth of water recorded in test boring is measured from top of ground to top of water level. Initial depth indicates water level during boring, completion depth indicates water level immediately after boring, and depth of "X" number hours indicates water level after letting water rise or fall over a time period. Water observations in pervious soil are considered reliable ground water levels for that date. Water observations in impervious soils can not be considered accurate ground water measurements for that date unless records are made over several days' time. Factors such as weather, soil porosity, etc., will cause the ground water level to fluctuate for both pervious and impervious soils.

SOIL DESCRIPTION

COLOR

When the color of the soil is uniform throughout, the color recorded will be such as brown, gray, black and may be modified by adjectives such as light and dark. If the soil's predominant color is shaded by a secondary color, the secondary color precedes the primary color, such as: gray-brown, yellow-brown. If two major and distinct colors are swirled throughout the soil, the colors will be modified by the term mottled, such as: mottled brown and gray.

PARTICLE SIZE	VISUAL	SOIL COMPONENTS	
Boulders	Larger than 8"	**Major Component**	**Minor Component Term**
Cobbles	8" to 3"	Gravel	Trace 1-10%
Gravel—Coarse	3" to ¾"	Sand	Some 11-35%
Fine	2 mm. To ¾"	Silt	And 36-50%
Sand —Coarse	2 mm.-0.6 mm. (Pencil lead size)	Clay	
—Medium	0.6 mm.-0.2 mm.	**Moisture Content**	
	(Table sugar and salt size)	**Term**	**Relative Moisture**
—Fine	0.2 mm.-0.06 mm.	Dry	Powdery
	(Powdered sugar and human hair size)	Damp	Moisture content below plastic limit
Silt	0.06 mm.-0.002 mm.	Moist	Moisture content
Clay	0.002 and smaller (Particle size of both Silt and Clay not visible		above plastic limit but below liquid limit
	to naked eye)	Wet	Moisture content above liquid limit

Condition of Soil Relative to Compactness Granular Material		**Condition of Soil Relative to Consistency Cohesive Material**	
Very Loose	5 blows/ft. or less	Very Soft	3 blows/ft. or less
Loose	6 to 10 blows/ft.	Soft	4 to 5 blows/ft.
Medium Dense	11 to 30 blows/ft.	Medium Stiff	6 to 10 blows/ft.
Dense	31 to 50 blows/ft.	Stiff	11 to 15 blows/ft.
Very Dense	51 blows/ft. or more	Very Stiff	16 to 30 blows/ft.
		Hard	31 blows/ft. or more

STANDARD PENETRATION RESISTANCE (ASTM D1586)

The purpose of this test is to determine the relative consistency of the soils in a boring, or from boring to boring over the site. This method consists of making a hole in the ground and driving a 2 inch O.D. split spoon sampler into the soil with a 140 pound hammer dropped from a height of 30 inches. The sampler is driven 18 inches and the number of blows recorded for each 6 inches of penetration. Values of standard penetration (N) are determined in blows per foot, summarizing the blows required for the last two 6 inch increments of penetration. (Example: 2-6-8; N = 14)

THIN-WALLED SAMPLER (ASTM D1587)

The purpose of the thin-walled sampler is to recover a relatively undisturbed soil sample for laboratory tests. The sampler is a thin-walled seamless tube with a 3 inch outside diameter, which is hydraulically pressed into the ground, at a constant rate. The ends are then sealed to prevent moisture loss, and the tube is returned to the laboratory for tests.

UNCONFINED COMPRESSION OR TRIAXIAL TESTS (ASTM D2166)

The unconfined compression test and the triaxial tests are performed to determine the shearing strength of the soil, to use in establishing its safe bearing capacity. In order to perform the unconfined compression tests, it is necessary that the soil exhibit sufficient cohesion to stand in an unsupported cylinder. These tests are normally performed on samples which are 6.0 inches in height and 2.85 inches in diameter. In the triaxial test, various lateral stresses can be applied to more closely simulate the actual field conditions. There are several different types of triaxial tests. These are, however, normally performed on constant strain apparatus with a deformation rate of 0.05 inches per minute.

CONSOLIDATION TEST (ASTM D2435)

The purpose of this test is to determine the compressibility of the soil. This test is performed on a sample of soil which is 2.5 inches in diameter and 1.0 inch in height, and has been trimmed from relatively "undisturbed" samples. The test is performed with a level system or an air activated piston for applying load. The loads are applied in increments and allowed to remain on the sample for a period of 24 hours. The consolidation of the sample under each individual load is measured and a curve of void ratio vs. Pressure is obtained. From the information obtained in this manner and the column loads of the structure, it is possible to calculate the settlement of each individual building column. This information, together with the shearing strength of the soil, is used to determine the safe bearing capacity for a particular structure.

REVISED TO ASTM D4318
ATTERBERG LIMITS (ASTM D423 AND D424)

These tests determine the liquid and plastic limits of soils having a predominant percentage of fine particle (silt and clay) sizes. The liquid limit of a soil is the moisture content expressed as a percent at which the soil changes from a liquid to a plastic state, and the plastic limit is the moisture content at which the soil changes from a plastic to a semi-solid state. Their difference is defined as the plasticity index (P.I. = L.L. - P.L.), which is the change in moisture content required to change the soil from a "semi-solid" to a liquid. These tests furnish information about the soil properties which is important in determining their relative swelling potential and their classifications.

MECHANICAL ANALYSIS (ASTM D422)

This test determines the percent of each particle size of a soil. A sieve analysis is conducted on particle sizes greater than a No. 20 sieve (0.074 mm), and a hydrometer test on particles smaller than the No. 200 sieve. The gradation curve is drawn through the points of cumulative per cent of particle size, and plotted on semi-logarithmic paper for the combined sieve and hydrometer analysis. This test, together with the Atterberg Limits tests, is used to classify a soil.

NATURAL MOISTURE CONTENT (ASTM D2216)

The purpose of this test is to indicate the range of moisture contents present in the soil. A wet sample is weighed, placed in the constant temperature oven at 105° for 24 hours, and re-weighed. The moisture content is the change in weight divided by the dry weight.

PROCTOR TESTS

The purpose of these tests is to determine the maximum density and optimum moisture content of a soil. The Modified Proctor test is performed in accordance with ASTM D1557-70. The test is performed by dropping a 10 pound hammer 25 times from an 18 inch height on each of 5 equal layers of soil in a 1/30 cubic foot mold, which represents a compaction effort of 56,250 foot pounds per cubic foot. The moisture content is then raised, and this procedure is repeated. A moisture density curve is then plotted, with the density on the ordinate axis and the moisture content on the abscissa axis. The moisture content at which the maximum density requirement can be achieved with a minimum compactive effort is designated as the optimum moisture content (O.M.C.). The Standard Proctor test is performed in accordance with ASTM D698-70. This test is similar to the Modified Proctor test and is performed by dropping a 5.5 pound hammer 25 times from a height of 12 inches on 3 equal layers of soil in a 1/30 cubic foot mold, which represents a compaction effort of 12,375 foot pounds per cubic foot. This test gives proportionately lower results than the Modified Proctor test.

FIELD CLASSIFICATION SYSTEM FOR ROCK EXPLORATION

Sarpolite A transitional material between soil and rock retains the relic structure of the parent rock and exhibits penetration resistance between 60 blows per foot and 100 blows/2 inches of penetration.

R.Q.D. Rock Quality Designation; Ratio of the core lengths greater than four inches to the total length of the core run.

Description	Percentage Core Recovered	RQD Rock Quality Description	Description of Rock Quality
Incompetent	Less than 40	0 - 25	very poor
Competent	40 - 70	25 - 50	poor
Fairly Competent	70 - 80	50 - 75	fair
Fairly Continuous	80 - 90	75 - 90	good
Continuous	90 - 100	90 - 100	excellent

FIELD HARDNESS: (A measure of resistance to scratching or abrasion)

Very Hard	Cannot be scratched with knife or sharp pick, breaking of hand specimens requires several hard blows of geologist's pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of a hammer required to detach hand specimen.
Moderately Hard	Can be scratched with knife or pick. Gouges or grooves to ¼ inch deep can be excavated by hard blow of point of a geologist's pick. Hand specimens can be detached by moderate blow.
Medium	Can be grooved or gouged 1/16 inch deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1 inch maximum size by hard blows of the point of a geologist's pick.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips and pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.
Very soft	Can be carved with knife. Can be excavated with point of pick. Pieces 1 inch or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

WEATHERING: (The action of the elements in altering the color, texture, and composition of the rock)

Very slightly	Rock generally fresh, joints stained, some joints may contain thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slightly	Rock generally fresh, joins stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.
Moderately	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some may be decomposed to clay. Rock as dull sound under hammer and has a significant loss of strength compared with fresh rock.
Severely	All rock except quartz discolored or stained. Rock "fabric" clear and evident but reduced in strength to strong soil. In granitoid rocks all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
Very severely	All rock except quartz discolored of stained. Rock "fabric" discernible, but mass effectively reduces to "soil" with only fragments of strong rock usually left.
Completely	All rock completely altered to soil-like material.

ROCK FRACTURE FREQUENCY: (Any break in a rock whether or not it has undergone relative displacement.)

Description	Spacing Between Fractures
Extremely fractured	Less than 1 inch
Moderately fractured	1 inch to 4 inches
Slightly fractured	4 inches to 8 inches
Sound	More than 8 inches

Note: Fracture frequency terms are generalized to described the average condition of the rock obtained from the core run. Portions of the rock within the run described may vary from the generalized descriptions. Where a core break appears to be due to drilling and not to natural causes, it has not been considered as a break for accessing fracture frequency. Frequency shown on Record of Soil Exploration represents condition of core as removed form the core barrel.

JOINTS BEDDING, AND FOLIATION:

Joints	Bedding & Foliation	Spacing
Very close	Very thin	Less than 2 inches
Close	Thin	2 inches - 1 foot
Moderately close	Medium	1 foot - 3 feet
Wide	Thick	3 feet - 10 feet
Very wide	Very Thick	More than 10 feet

Notes: Refers to perpendicular distance between discontinuities

Attitude	Angle (degrees)
Horizontal	0 to 5
Shallow to low angle	5 to 35
Moderately dipping	35 to 55
Steep or high angle	55 to 85
Vertical	85 to 90

CBC Engineers & Associates, Ltd.
125 Westpark Road
Centerville, OH 45459
(P) (937) 428-6150 / (F) (937) 428-6154

BORING LOG

CLIENT: Clark County Engineer's Department | REPORT NO.: 10731 | BORING NO.: B-1
DATE STD.: 6/28/09 | DATE FINISHED: 6/29/09
PROJECT: Redmond Rd., Springfield, OH | DRILLERS: CBC | GROUND ELEV.: 1016.13
LOCATION: As Shown on the Boring Location Plan | METHOD: 3 ¼ HSA

SCALE, FT.	STRATUM DEPTH, FT.	CLASSIFICATION OF MATERIAL Major Soil Components: Gravel, Sand, Silt, Clay Minor Component Term: Trace 1-10%, Some 11-35%, And 36-50%	SAMPLE NUMBER & SAMPLE TYPE	DEPTH OF SAMPLE, FT. FROM	TO	BLOWS ON SAMPLER PER SPT (6" INTER-VAL)	SPT "N", OR RECOVERY (IN. FOR SHELBY TUBES, % FOR ROCK CORE)
0.0	0.0	FILL, brown to black SAND, GRAVEL, SILT, CLAY (moist)					
			1A	1.0	2.5	5-9-5	14
			2A	3.5	5.0	5-4-5	9
			3A	6.0	7.5	2-3-3	6
	8.0	ORIGINAL, medium stiff, brown CLAY, some silt, fine sand (moist) (alluvium)	4A	8.5	10.0	2-2-7	9
10.0							
	12.0	Very stiff gray SILT, some clay, some sand, trace gravel (damp) (glacial till)					
			5A	13.5	15.0	7-11-11	22
		Becomes hard at 18.5'	6A	18.5	20.0	10-16-26	42
20.0							
			7A	23.5	25.0	16-18-21	39
			8A	28.5	30.0	17-17-20	37
30.0							
		Becomes very stiff at 33.5'					
			9A	33.5	35.0	16-15-15	30
			10A	38.5	40.0	15-16-18	34
40.0							
		Becomes hard at 43.5'	11A	43.5	45.0	13-13-20	33
			12A	48.5	50.0	22-25-28	53
50.0							
		BOTTOM OF BORING AT 50.0'					
60.0							
70.0							

WATER LEVEL OBSERVATIONS
Noted on rods DRY ft.
At completion DRY ft.
After -- hours -- ft.

BORING METHOD
HSA Hollow Stem Auger
CFA Continuous Flight Auger
DC Driven Casing
MD Mud Drilling
RC Rock Coring
CA Casing Advancer

TYPE SAMPLE
A - Split Spoon
B - Rock Core
C - Shelby Tube
D - Other

*These Shelby Tube Samples Obtained In An Auxiliary Boring Drilled A Few Feet From This Boring

CBC Engineers & Associates, Ltd.
125 Westpark Road
Centerville, OH 45459
(P) (937) 428-6150 / (F) (937) 428-6154

BORING LOG

CLIENT: Clark County Engineer's Department
PROJECT: Redmond Rd., Springfield, OH
LOCATION: As Shown on the Boring Location Plan

REPORT NO.: 10731
DATE STD.: 6/28/09
DRILLERS: CBC
METHOD: 3 ¼ HSA

BORING NO.: B-2
DATE FINISHED: 6/29/09
GROUND ELEV.: 1016.59

SCALE, FT.	STRATUM DEPTH, FT.	CLASSIFICATION OF MATERIAL Major Soil Components: Gravel, Silt, Sand, Clay Minor Component Term: Trace 1-10%, Some 11-35%, And 36-50%	SAMPLE NUMBER & SAMPLE TYPE	DEPTH OF SAMPLE, FT. FROM	DEPTH OF SAMPLE, FT. TO	BLOWS ON SAMPLER PER SPT (6" INTER-VAL)	SPT "N", OR RECOVERY (IN. FOR SHELBY TUBES, % FOR ROCK CORE)
0.0	0.0	FILL, brown to black SAND, GRAVEL, SILT, CLAY (moist)	1A	1.0	2.5	7-5-5	10
			2A	3.5	5.0	3-3-4	7
	6.0	ORIGINAL, medium stiff brown CLAY, some silt, fine sand (moist) (alluvium)	3A	6.0	7.5	4-5-4	9
	8.0	Medium dense brown SAND, some gravel trace silt (damp)	4A	8.5	10.0	5-5-7	12
10.0							
	12.0	Very stiff gray SILT, some clay, some sand, trace gravel (damp) (glacial till)	5A	13.5	15.0	12-12-18	30
		Becomes hard at 18.5'	6A	18.5	20.0	5-11-20	31
20.0							
			7A	23.5	25.0	5-11-20	31
			8A	28.5	30.0	12-14-18	32
30.0							
		Becomes very stiff at 33.5'	9A	33.5	35.0	16-11-11	22
			10A	38.5	40.0	12-12-12	24
40.0							
		Becomes hard at 43.5'	11A	43.5	45.0	9-15-16	31
			12A	48.5	50.0	21-22-26	48
50.0		BOTTOM OF BORING AT 50.0'					
60.0							
70.0							

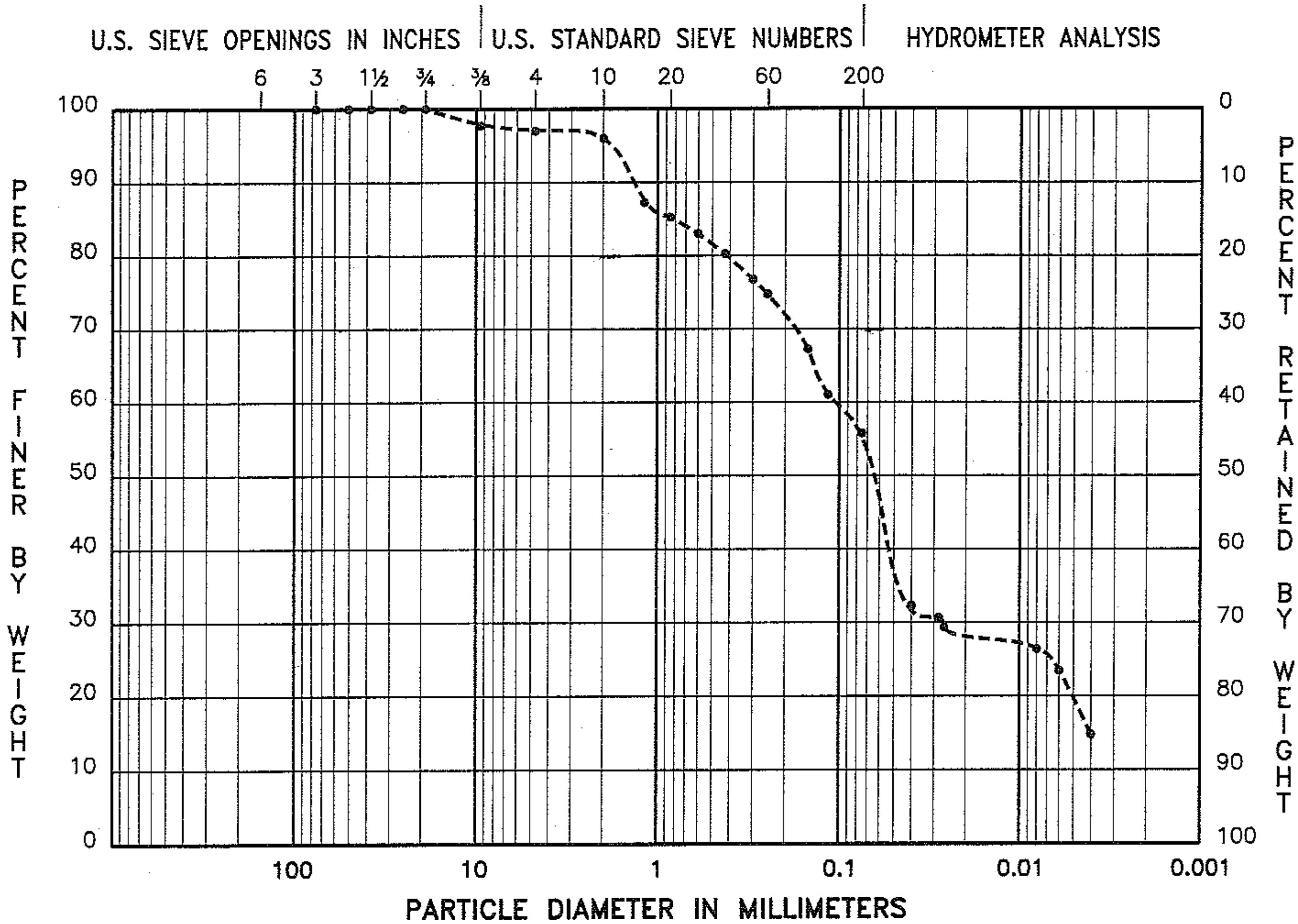
WATER LEVEL OBSERVATIONS
Noted on rods DRY ft.
At completion DRY ft.
After -- hours -- ft.

BORING METHOD
HSA Hollow Stem Auger
CFA Continuous Flight Auger
DC Driven Casing
MD Mud Drilling
RC Rock Coring
CA Casing Advancer

TYPE SAMPLE
A - Split Spoon
B - Rock Core
C - Shelby Tube
D - Other

*These Shelby Tube Samples Obtained In An Auxiliary Boring Drilled A Few Feet From This Boring

GRAIN SIZE DISTRIBUTION TEST REPORT



COBBLES	GRAVEL		SAND		SILT	CLAY
	coarse	fine	coarse	fine		

	Specimen Identification	Material Description	MC%	LL	PL	PI	Cc	Cu
●		–						
		–						

	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	%Gravel	%Sand	%Silt	%Clay
●	0.83mm	0.105mm	0.06mm	0.026mm	0.004mm	–	3.93	40.27	41.05	14.75

Project: CBC–10731
TRUSS BRIDGE ON REDMOND BRIDGE
SPRINGFIELD, OHIO

Date: 07/02/09

CBC ENGINEERS
DAYTON, OHIO

Remarks:

GRAIN SIZE ANALYSIS WITH HYDROMETER WORKSHEET
ASTM D 422

Project No.: 10731 **Name:** BILL ROBERTSON

Sample Data:

Sample No.:	1	Hydrometer Jar No.:	1
Sample Description:	GLACIAL TILL	Large Particle Dia. (in.):	3/8"
Sample Location:	B-2	Min. Sample Size (gm):	500
Sample Depth: (FT)	13.5 45.0	Min. Sample ‹ No. 10 (gm):	50
Hydrometer Type:	151H	Total Sample Mass (gm):	733.6
Soil Spec. Gravity	2.69	Hydro. Sample Mass (gm):	58.6

Hygroscopic Moisture Correction:

Tare No.:	1
Tare Wt. (gm):	3
Wet Soil + Tare:	19.7
Dry Soil + Tare:	19.6
Correction Factor:	0.994012

Grain Size Data, Material ≥ No. 10 Sieve:

Sieve	Mass Ret. (gm)	% Finer (%)	
3-in.	0.00	100.000	
2-in.	0.00	100.000	
1 1/2 in.	0.00	100.000	
1 in.	0.00	100.000	
3/4 in.	0.00	100.000	
3/8 in.	16.90	97.696	3.908
No. 4	21.90	97.015	3.88
No. 10	28.80	96.074	3.84
Tot:	**67.60**		

Small Fraction Grain Size Data:

Sieve	Hyd. Ret. (gm)	Amt. Ret (gm)	% Finer (%)	
No. 16	2.10	26.29	87.20	3.49
No. 20	1.10	13.77	85.32	3.41
No. 30	1.40	17.53	82.94	3.32
No. 40	1.50	18.78	80.38	3.21
No. 50	2.10	26.29	76.79	3.07
No. 60	1.20	15.02	74.74	2.99
No. 100	4.40	55.08	67.24	2.69
No. 140	3.70	46.32	60.92	2.44
No. 200	3.00	37.56	55.80	2.23

Hydrometer Fraction Grain Size Data:

Hyd Sample Mass (gm):	52.50
Oven Dry Mass (gm):	52.19
W (total soil mass) (gm):	54.32

T (min)	Temp (°C)	Hyd.	R	Hyd. Corr.	P	K	L	D
2	24	1.014	0.003	###	32.23	0.01267	12.6	0.03180
5	24	1.014	0.003	###	32.23	0.01267	12.6	0.02011
15	24	1.013	0.003	###	29.30	0.01267	12.9	0.01175
30	24	1.012	0.003	###	26.37	0.01267	13.1	0.00837
60	24	1.011	0.003	###	23.44	0.01267	13.4	0.00599
250	24	1.010	0.003	###	20.51	0.01267	13.7	0.00297
				###	######			#DIV/0!

Graphing Data:

Size (mm)	% Finer (%)	
76.2	100.000	
50.8	100.000	
38.1	100.000	
25.4	100.000	
19.05	100.000	
9.525	97.696	3.908
4.75	97.015	3.88
2	96.074	3.84
1.18	87.20	3.49
0.85	85.32	3.41
0.6	82.94	3.32
0.425	80.38	3.21
0.3	76.79	3.07
0.25	74.74	2.99
0.15	67.24	2.69
0.106	60.92	2.44
0.075	55.80	2.23
0.031801	32.23398	1.29
0.020113	32.23398	1.29
0.01175	29.30362	1.172
0.008372	26.37326	1.055
0.005988	23.44289	.938
0.002966	20.51253	.820
#DIV/0!	########	

21.85

Hydrometer Effective Depth "L"

Hyd. Rdg.	L	Hyd. Rdg.	L
1.000	16.3	1.020	11.0
1.001	16.0	1.021	10.7
1.002	15.8	1.022	10.5
1.003	15.5	1.023	10.2
1.004	15.2	1.024	10.0
1.005	15.0	1.025	9.7
1.006	14.7	1.026	9.4
1.007	14.4	1.027	9.2
1.008	14.2	1.028	8.9
1.009	13.9	1.029	8.6
1.010	13.7	1.030	8.4
1.011	13.4	1.031	8.1
1.012	13.1	1.032	7.8
1.013	12.9	1.033	7.6
1.014	12.6	1.034	7.3
1.015	12.3	1.035	7.0
1.016	12.1	1.036	6.8
1.017	11.8	1.037	6.5
1.018	11.5	1.038	6.2
1.019	11.3		

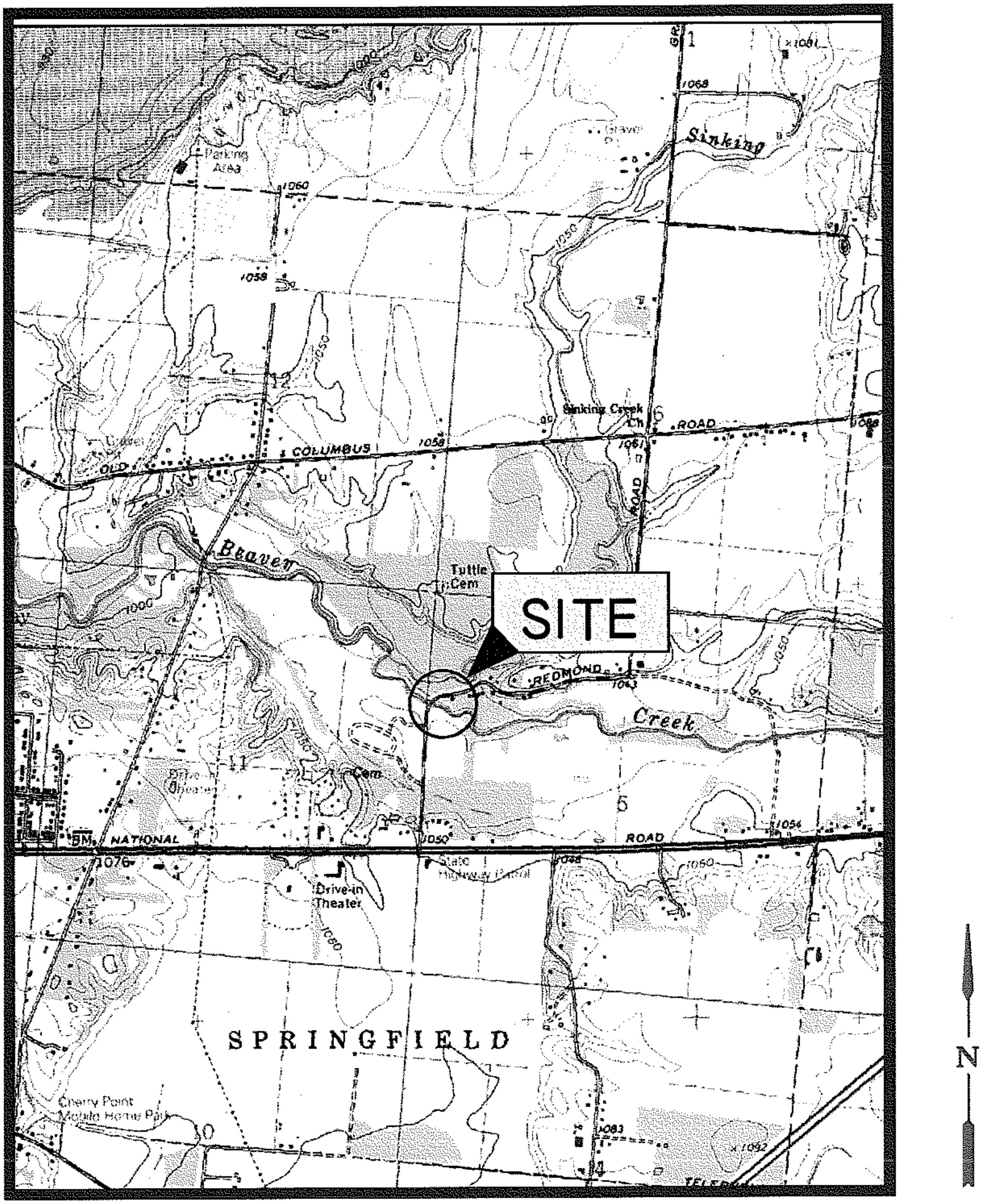
Hydrometer Correction Factors "R":

Temp (°C)	R
16	1.004
17	1.004
18	1.004
19	1.004
20	1.004
21	1.004
22	1.003
23	1.003
24	1.003
25	1.003

LABORATORY SUMMARY SHEET

BORING NO.	DEPTH INCREMENT, (FT.)	NATURAL MOISTURE CONTENT, %
B-1	1.0 – 2.5	6.7
B-1	3.5 – 5.0	15.0
B-1	6.0 – 7.5	26.7
B-1	8.5 – 10.0	25.4
B-1	13.5 – 15.0	10.2
B-1	18.5 – 20.0	9.2
B-1	23.5 – 25.0	10.0
B-1	28.5 – 30.0	10.0
B-1	33.5 – 35.0	10.8
B-1	38.5 – 40.0	10.6
B-1	43.5 – 45.0	10.6
B-1	48.5 – 50.0	9.2
B-2	1.0 – 2.5	13.8
B-2	3.5 – 5.0	14.2
B-2	6.0 – 7.5	17.2
B-2	8.5 – 10.0	14.8
B-2	13.5 – 15.0	10.2
B-2	18.5 – 20.0	10.4
B-2	23.5 – 25.0	9.9
B-2	28.5 – 30.0	9.9
B-2	33.5 – 35.0	10.9
B-2	38.5 – 40.0	10.4
B-2	43.5 – 45.0	10.6
B-2	48.5 – 50.0	4.9

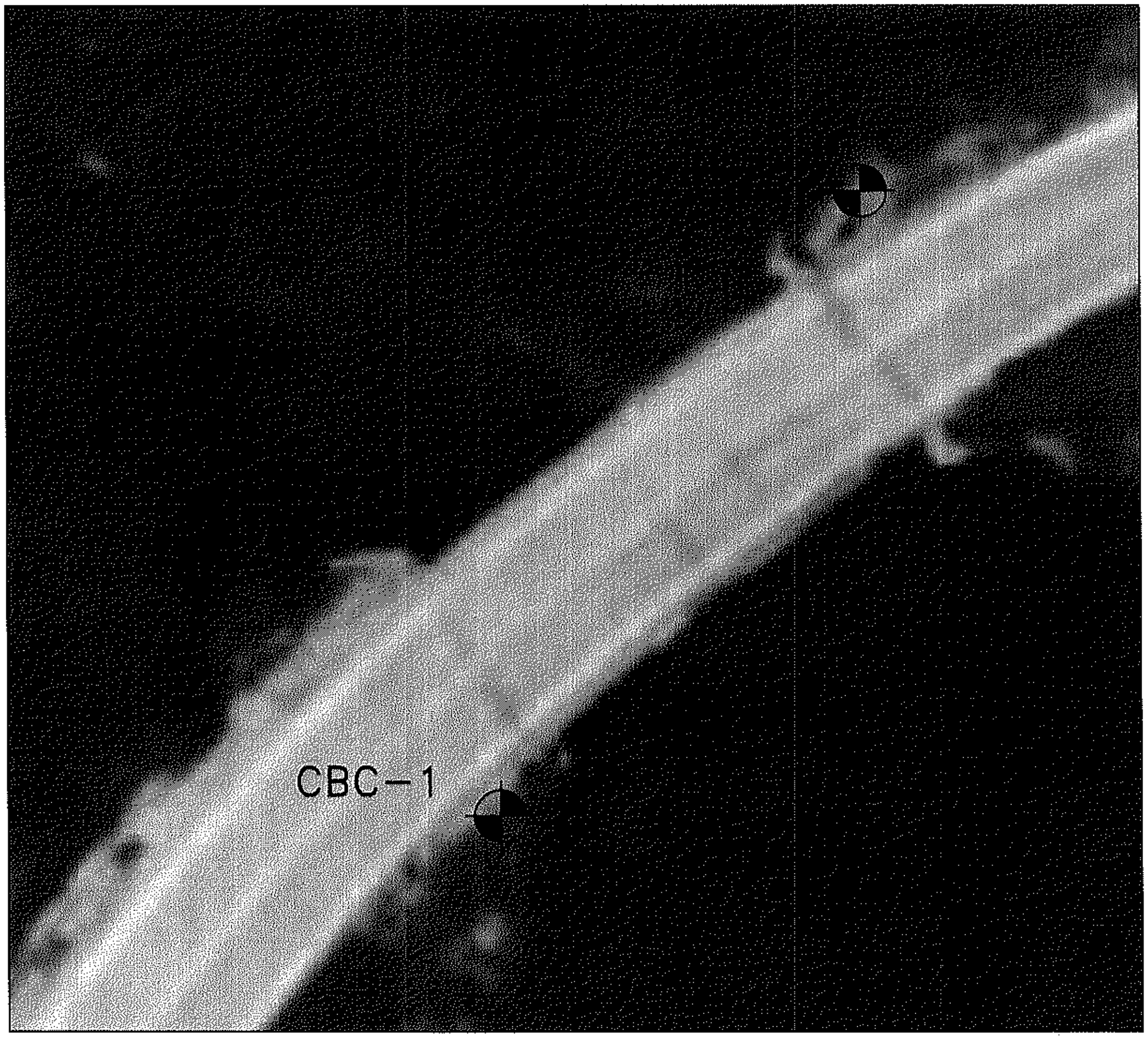
NEW MOOREFIELD QUADRANGLE



VICINITY MAP

GEOTECHNICAL ENGINEERING INVESTIGATION
FOR A PROPOSED REPLACEMENT BRIDGE
FOR A EXISTING TRUSS BRIDGE ON REDMOND ROAD
SPRINGFIELD, OHIO

PROJECT NO. CBC-10731	CBC ENGINEERS DAYTON, OHIO
SCALE 1" = 2000'	
FIGURE NO.	1



N

LEGEND

CBC-1 = BORING LOCATION

BORING LOCATION PLAN

GEOTECHNICAL ENGINEERING INVESTIGATION
FOR A PROPOSED REPLACEMENT BRIDGE
FOR A EXISTING TRUSS BRIDGE ON REDMOND ROAD
SPRINFIELD, OHIO

PROJECT NO. CBC-10731	CBC ENGINEERS DAYTON, OHIO
SCALE N.T.S.	
FIGURE NO.	2